Simulation of Flow and Evaluation of Bridge Scour at Horse Island Chute Bridge near Chester, Illinois

Water-Resources Investigations Report 01–4176

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U.S. GEOLOGICAL SURVEY

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ABSTRACT

The evaluation of scour at bridges throughout the State of Missouri has been ongoing since 1991, and most of these evaluations have used one-dimensional hydraulic analysis and application of conventional scour depth equations. Occasionally, the conditions of a site dictate that a more thorough hydraulic assessment is required. To provide the hydraulic parameters required to determine the potential scour depths at the bridge over Horse Island Chute near Chester, Illinois, a two-dimensional finite-element surface-water model was used to simulate flood flows in the vicinity of the Missouri State Highway 51 crossing of the Mississippi River and Horse Island Chute.

The model was calibrated using flood-flow information collected during the 1993 flood. A flood profile along the Illinois side of the Mississippi River on August 5, 1993, with a corresponding measured discharge of 944,000 cubic feet per second was used to calibrate the model. Two additional flood-flow simulations were run: the flood peak that occurred on August 6, 1993, with a maximum discharge of 1,000,000 cubic feet per second, and the discharge that caused impending overtopping of the road embankment in the vicinity of the Horse Island Chute bridge, with a discharge of 894,000 cubic feet per second (impendent discharge).

Hydraulic flow parameters obtained from the simulations were applied to scour depth equations to determine general contraction and local pier and abutment scour depths at the Horse Island Chute bridge. The measured discharge of 944,000 cubic feet per second resulted in 13.3 feet of total combined contraction and local pier scour at Horse Island Chute bridge. The maximum discharge of 1,000,000 cubic feet per second resulted in 15.8 feet of total scour and the impendent discharge of 894,000 cubic feet per second resulted in 11.6 feet of total scour.

INTRODUCTION

In 1991, the U.S. Geological Survey (USGS) began a study in cooperation with the Missouri Department of Transportation (MoDOT) to analyze and assess nearly 3,300 bridges throughout Missouri for scour susceptibility. Approximately 270 bridges throughout the state were identified as "scour-susceptible," requiring a detailed hydraulic evaluation to determine potential scour depths. The term "scour-susceptible" describes a bridge that is deemed potentially unstable because abutment or pier foundations, or both, have the potential to be undermined because of erosion to the channel bed or banks (Federal Highway Administration, 1988). Most of these bridges have been assessed using techniques outlined in Hydraulic Engineering Circular No. 18 (Richardson and others, 1995), which includes a one-dimensional hydraulic analysis and application of predictive scour-depth computations.

Occasionally, the simplifying assumptions that must be made to apply a one-dimensional hydraulic analysis to a two-dimensional flow scenario are deemed unreasonable. This is the case at structure L 1004R1 on Missouri State Highway 51, crossing Horse Island Chute in Perry County, Missouri, near Chester, Illinois, hereinafter referred to as Horse Island Chute bridge. Horse Island Chute bridge (fig. 1) is built on a horizontal curve, and is near a bridge over the Missis-

Figure 1. Horse Island Chute bridge (structure L 1004R1) on Missouri State Highway 51, in Perry County, Missouri, near Chester, Illinois, as viewed from the Bois Brule Levee on the upstream right (southwest) floodplain. The Mississippi River bridge (structure L 135A) and bluff on the Illinois side of the Mississippi River are in the background.

sippi River (structure L 135A, hereinafter referred to as Mississippi River bridge) on a constricted section of the floodplain bounded by the Bois Brule Levee on the right floodplain (as viewed facing downstream) and a high bluff on the Illinois side of the Mississippi River. This unique configuration is beyond the bounds of reasonability for a one-dimensional hydraulic analysis, and requires a two-dimensional hydraulic analysis of the site.

The flood of 1993 in the central United States—recognized as one of the most dramatic and devastating floods in U.S. history—occurred in the Lower Missouri and Upper Mississippi River Basins, which includes the Mississippi River and Horse Island Chute at Missouri State Highway 51. Commonly, the hydraulic analysis of a scour-susceptible site must proceed without the benefit of major flood information, against which the hydraulic model of the site can be calibrated. However, the duration of the flood and the flow scenario at the site before, during, and after the peak of the flood on August 6, 1993, along with documented flood information, provided nearly ideal hydraulic conditions for the determination of worst-case scour depths at Horse Island Chute bridge. The

data collected at the USGS streamflow-gaging station on the Mississippi River at Chester, Illinois (07020500) were available for use in calibrating the two-dimensional hydraulic model at this site. At the gaging station, the Mississippi River had a flood-peak elevation of 390.79 ft (feet) above sea level on August 6, 1993, which exceeded the April 1973 flood-peak elevation by more than 6 ft.

Purpose and Scope

This report gives the results of a study in which the numerical, two-dimensional, Finite Element Surface-Water Modeling System (FESWMS-2DH, Froehlich, 1989) was used to determine the hydraulic parameters needed for an evaluation of the contraction and local pier and abutment scour that might occur at Horse Island Chute bridge on Missouri State Highway 51 in Perry County, Missouri, near Chester, Illinois. The report also documents the flow distribution and velocities of floodwaters in the Mississippi River, Horse Island Chute, and associated floodplains for a discharge measurement on August 5, 1993 (hereinafter

referred to as measured discharge), the flood-peak discharge on August 6, 1993 (hereinafter referred to as maximum discharge), and the discharge that causes impendent overtopping of Missouri State Highway 51 (hereinafter referred to as impendent discharge).

Description of Study Reach

The study reach begins near river mile 108 and ends near river mile 111 on the Mississippi River near Chester, Illinois (fig. 2). The total width of the Mississippi River floodplain throughout this reach generally is about 5 mi (miles), but is dissected by various levee districts. Upstream from the study reach, the Mississippi River is isolated from the Old River channel by Kaskaskia Island. The Old River floodway is confined between the northwestern edge of the Bois Brule Levee and the southeastern edge of the Kaskaskia Levee. Horse Island Chute splits from the Old River approximately 3,000 ft upstream from the mouth of the Old River (approximately 3,000 ft upstream from the Missouri State Highway 51 bridge over the Mississippi) and flows into the Mississippi River approximately 1,400 ft downstream from the Mississippi River bridge. Horse Island is bounded by the Mississippi River on the northeast, the Old River on the northwest, and Horse Island Chute on the south. Near the point where Horse Island Chute separates from the Old River, the Bois Brule Levee trends toward the east, parallel to Horse Island Chute, and approaches the southern bank of the Mississippi River. The Bois Brule Levee then turns toward the southeast and essentially follows the southern bank of the Mississippi River for several miles. The Bois Brule Levee creates a constriction on the floodplain of the Mississippi River that narrows to a minimum width of 2,230 ft approximately 3,500 ft downstream from the Mississippi River bridge. During the 1993 flood, the Bois Brule Levee was not overtopped in the study reach; thus, the study reach was constrained between the Bois Brule Levee on the Missouri side and the railroad embankment along the toe of the bluffs on the Illinois side. A shaded relief image of the study reach in an oblique perspective facing downstream is shown in figure 3.

Missouri State Highway 51 crosses the Mississippi River in a northeast direction between Bois Brule Levee and the Illinois bluffs near river mile 110, approximately 8,400 ft upstream from the downstream boundary of the study reach. Missouri State Highway 51 is bearing to the northwest inside (south of) the Bois Brule Levee and begins to curve to the northeast as it crosses the levee. The Missouri State Highway 51 curve continues as it crosses Horse Island Chute and terminates just before the Mississippi River bridge (figs. 2 and 3). The Horse Island Chute bridge (structure L 1004R1) is 464 ft long and was built on a horizontal curve (fig. 2). The Mississippi River bridge (structure L 135A) is 2,827 ft long and is raised substantially above normal water-surface elevations for barge traffic on the Mississippi River. A short section of raised road embankment exists between the southern end of the Horse Island Chute bridge and the Bois Brule Levee (fig. 3). Another section of raised and curved road embankment extends northward from the Horse Island Chute bridge, rising to meet the sloped approach spans of the Mississippi River bridge.

Description of the 1993 Flood

A USGS streamflow-gaging station has been collecting continuous record on the Mississippi River at Chester (station 07020500; fig. 2) since October 1927. As the floodwaters rose during July 1993, several levees in the Mississippi River floodplains were breached, often resulting in flood levels that were temporarily lowered in the vicinity of the breach and upstream of the breach. The Bois Brule Levee, in combination with the raised and super-elevated road embankment of Missouri State Highway 51, permitted access to at least one lane of traffic across the Mississippi River floodwaters into late July. However, on July 22, 1993, a breach in the Bois Brule Levee occurred approximately 12 river miles downstream from the Mississippi River bridge. The river level upstream of the breach and at the gage dropped noticeably for a period of time as the area behind the levee flooded; however, Missouri State Highway 51 had to be closed because of inundation of the highway inside the levee.

The floodwaters again continued to rise until a flood-peak elevation of 390.79 ft above sea level was measured at the gage on August 6, 1993. This peak elevation corresponds to a discharge of 1,000,000 ft^3/s (cubic feet per second). The 1993 peak surpassed the previous flood-peak elevation of 380.85 ft on June 30, 1844, but was slightly less than the 1844 peak discharge of 1,050,000 ft^3/s. The constricting effect of the
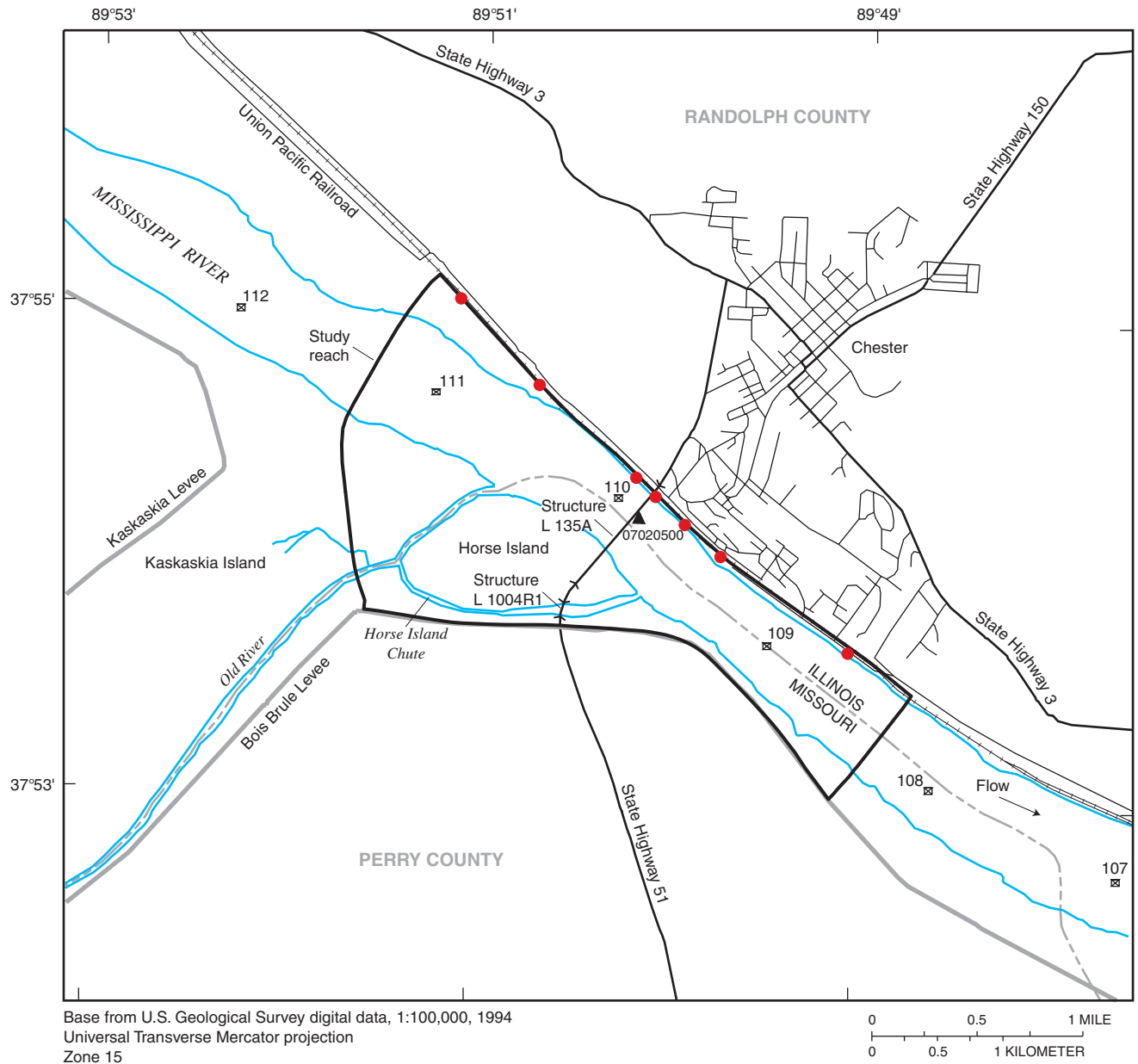
Figure 2. Location of the study reach near Chester, Illinois.

EXPLANATION

⊠ 112   RIVER-MILE MARKER—Number is distance upstream
         from the junction with the Ohio River, in miles

▲ 07020500   U.S. GEOLOGICAL SURVEY STREAMFLOW-GAGING
             STATION AND NUMBER

● LOCATION OF MEASURED WATER-SURFACE ELEVATION
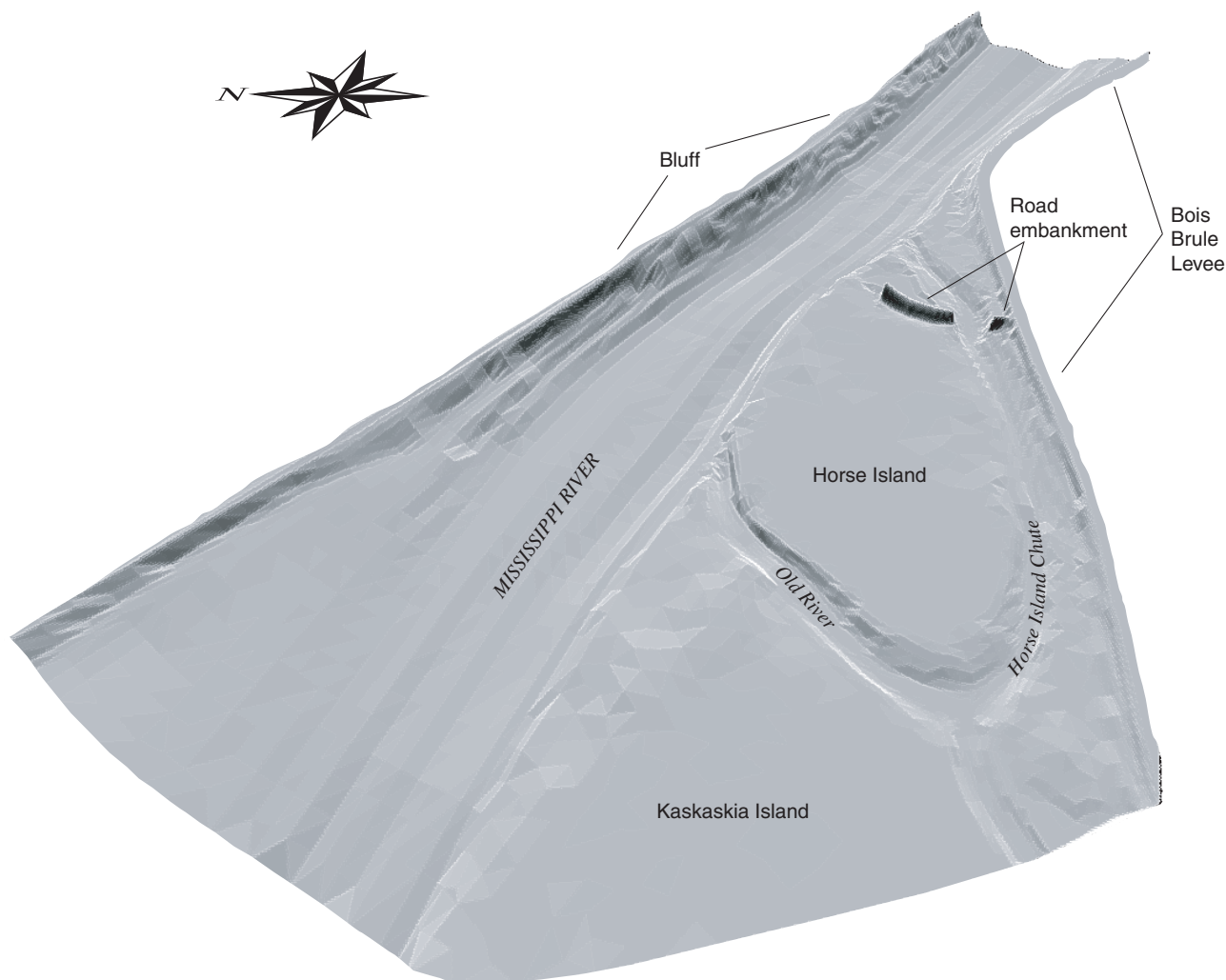 FROM FLOOD PROFILE ON AUGUST 5, 1993

Figure 3. Shaded relief image of study reach near Chester, Illinois, in an oblique perspective facing downstream.

Bois Brule Levee, which was not present in 1844, caused the 1993 peak elevation to be larger than the peak elevation in 1844, but caused the peak discharge to be less.

On August 5, 1993, prior to the peak, a flood profile was obtained along the railroad embankment on the Illinois side of the river from approximately 12,200 ft upstream from the Mississippi River bridge to 15,280 ft downstream (fig. 2). This profile indicated a river surface slope of 1.7 ft/mi (feet per mile) on August 5, 1993. An Acoustic Doppler Current Profiler (ADCP) measurement made concurrent with the flood profile downstream of the bridges and road embankment indi-

cated the discharge at that time was approximately 944,000 ft^3/s. Water was over Missouri State Highway 51 and Horse Island Chute bridge at that time.

SIMULATION OF FLOW

The FESWMS-2DH model simulates flows in two dimensions in the horizontal plane. It uses a finite-element mesh and the Galerkin finite-element method of solving three partial-differential equations representing conservation of mass and momentum (Froehlich, 1989). This two-dimensional model can simulate longitudinal and lateral variations in water-surface ele-

vations and velocities and can accommodate geometric features such as highway embankments, channel bends, and bridge piers. A graphical user interface called Surface-Water Modeling System (SMS, Environmental Modeling Research Laboratory, 1999), was used for converting survey data into a scatter data set, developing the finite-element mesh from the scatter data, assigning roughness coefficients and other parameters to the mesh elements, and evaluating the FES-WMS-2DH model output.

Model Development

The finite-element mesh (fig. 4) was designed to closely represent the non-uniform characteristics of the study reach inundated by the 1993 Mississippi River flood at Chester. The mesh extended upstream and downstream from Missouri State Highway 51 to eliminate any errors in flow computations because of estimated boundary conditions. The reach begins about 8,400 ft downstream from the Mississippi River bridge, and continues to about 7,600 ft upstream from the bridge (fig. 4). The bluff along the left floodplain (as viewed facing downstream) determines the lateral extent of the mesh on the Illinois side, and the Bois Brule Levee determines the lateral extent of the mesh on the right floodplain (as viewed facing downstream) on the Missouri side. In November 1999 and January 2000, five cross sections were surveyed across the channel and floodplains using global positioning satellite (GPS) technology combined with conventional surveying techniques. These methods were used to correct, refine, and correlate topographic contours from a 1:24,000 scale digital raster graphic (DRG) image of the study area to define the topography of the reach.

In areas where substantial change in topography exists, the mesh was refined to accurately represent existing structures and channels, and to accommodate the model's sensitivity to wetting and drying of elements along boundaries. Elements became "wet" or "dry" as the water surface changes to cover or expose them. Examples of topographic features in this study reach that required refinement were levees, roadway embankments, and river banks. Furthermore, in areas where velocity, depth, and water-surface elevation changes were expected to be large, mesh details were increased to better facilitate simulation by the model. An example of this refinement is near the Horse Island Chute and Mississippi River bridges and the adjoining road embankments (fig. 4). The refinement also

improves the precision of the parameters needed for the scour computations at the Horse Island Chute bridge. The finite-element mesh used in this study has 6,576 elements. Each element has a node at each corner, at the mid-point of each side, and in the center, for a total of 22,060 nodes.

A particular flow scenario cannot be simulated directly because the simulation cannot be started with boundary conditions that exactly represent the true conditions. The true conditions that are to be simulated are reached in a process called "spindown." In a sub-critical flow reach (a reach where flow is deep and velocities are low, typical of large rivers), spindown involves starting the simulation with the desired discharge as the upstream boundary condition and a downstream tailwater elevation that is higher than the highest land-surface elevation in the mesh. This condition must be met so that all nodes and elements in the model are "wet," or have a positive depth of flow. The simulation is run with these conditions for a sufficient number of iterations to cause the water-level elevation changes between iterations to be minimized within a pre-set limit. The downstream tailwater elevation is then decreased by some finite amount, and the simulation is run again using the results of the previous run—called a "hot start"—as the starting point for the first iteration of the new run. This process is repeated until the desired tailwater elevation is reached, as dictated by high-water marks, flood profiles, or other known site parameters.

During the spindown process, if the simulated water-surface elevation at a particular node is less than the land-surface elevation assigned to the node, the node is said to "go dry." If one or more of the nodes for a particular element go dry, then the element goes "dry," and the element is not included in the computations during that iteration. As the simulation proceeds through the assigned number of iterations, it is possible for an element to oscillate between wet and dry; the user sets a tolerance on the depth of flow over a node. However, if an element goes dry and stays dry for several iterations, it should be disabled or the simulation may become unstable and not converge.

Calibration of Model to Observed Conditions

Typically, a model is calibrated to an extreme condition such as a flood peak, because flood-peak elevation marks can be identified after an event, and a steady-state condition is assumed. However, accurate

Figure 4. Finite-element mesh used in flow simulations near Chester, Illinois.

flood-peak elevations were not obtained in 1993 within the study reach. Elevations were estimated several months after the peak from high-water marks along the levees and generalized comments of local observers; however, accurate placement of the marks could not be established. For this study, a supplemental attempt to obtain additional flood-peak elevations in buildings along the bluff on the Illinois side resulted in very limited success. Therefore, the non-peak flood profile obtained on August 5, 1993, with a measured discharge of 944,000 ft^3/s and a water-surface elevation at the gage of 390.20 ft above sea level, were used as the calibration parameters.

A variety of land-use coverages were applied to elements in the mesh (fig. 5), and depth-dependent Manning's roughness coefficients (n-values) were assigned to each coverage type (table 1). In the depth-dependent n-value method, the "lower depth" n-value is applied when the water depth over the element is less than the lower depth, and the "upper depth" n-value is applied when the water depth is greater than the upper depth; when the water depth is between the lower and upper depths, the n-value is interpolated linearly from the upper and lower n-values (Froehlich, 1989). Depth-dependent n-values were used to account for changes in the roughness with increased depth of flow; for most land-use types, the roughness decreases with increased depth because the effect of the physical features causing the roughness decreases as the depth of flow increases (for example, grass and crops that lie over in high flows). For a few of the land-use types, the roughness increases with increased depth (for example, flow through overhanging tree branches in "Old River and Horse Island Chute" or the transition from stems and trunks to branches in "trees and brush"). The "spur dike" category was created to represent the effect of spur dikes along the banks; actual elevations of the dikes were unavailable, but the n-values assigned to the lower depth were increased to impede flow at lower depths of flow.

A discharge of 944,000 ft^3/s was used as the upstream boundary condition, and spindown of the model occurred until a tailwater elevation of 388.35 ft was reached at the downstream boundary. During calibration of the model, the roughness coefficients were adjusted within reasonable limits until simulated flood elevations satisfactorily matched the flood elevations determined during the profile on August 5, 1993 (fig. 6). The calibrated model results are within +/- (plus or minus) 0.20 ft of the surveyed flood elevations, except

the farthest upstream profile point and the profile points at the constriction (fig. 6). The larger differences at the constriction are thought to be the result of drawdown caused by the combined effects of the constriction and a shelf of sediment deposition at the mouth of Horse Island Chute during the channel survey in 1999, which presumably was not present during the 1993 flood. Simulated water-surface elevations from the measured discharge simulation are shown in figure 7. The results indicate the discharge through the Mississippi River bridge was 886,000 ft^3/s (93.9 percent of the measured discharge), and discharge through the Horse Island Chute bridge and over the road embankment between the bridges was 57,500 ft^3/s (6.1 percent of the measured discharge).

Simulation results also indicated that the maximum velocity in the Mississippi River was 11.5 ft/s (feet per second) and occurred at the levee constriction; the maximum depth was 79.2 ft and occurred near the Mississippi River bridge. The maximum velocity and depth in Horse Island Chute were 4.3 ft/s and 40.3 ft, and occurred near the bridge. The maximum velocity and depth over the road embankment were 2.6 ft/s and 3.9 ft.

Simulation of August 6, 1993, Maximum Discharge

The peak discharge of 1,000,000 ft^3/s was used as the upstream boundary condition for the maximum discharge simulation. The final results of the measured discharge simulation were used as the hot start for the first iteration of the maximum discharge simulation, and spindown of the simulation occurred until the tailwater elevation (the downstream boundary condition) approximately matched a flood-peak elevation mark obtained about 1,500 ft downstream from the downstream end of the reach. The same model configuration established in the calibration simulation was used in the maximum discharge simulation, except for a few elements near the north end of the road embankment that had been disabled (they had gone dry in the calibration simulation) and were re-enabled to permit flow because the water-surface elevations were higher than in the calibration simulation.

Simulated water-surface elevations from the maximum discharge simulation are shown in figure 8. The results indicate the discharge through the Mississippi River bridge was 936,000 ft^3/s (93.6 percent of the maximum discharge), and discharge through the
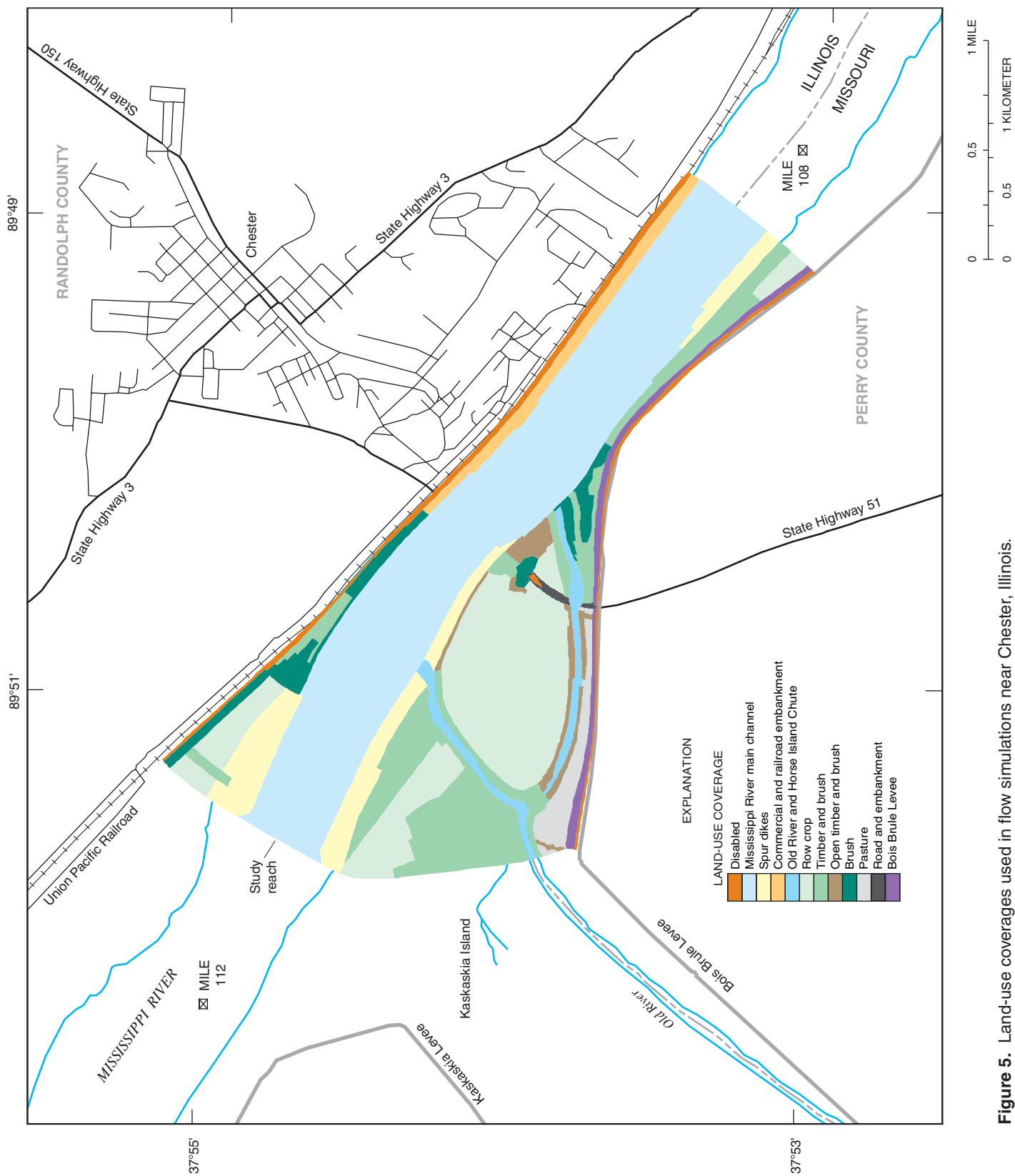
Figure 5. Land-use coverages used in flow simulations near Chester, Illinois.

EXPLANATION

LAND-USE COVERAGE

- Disabled
- Mississippi River main channel
- Spur dikes
- Commercial and railroad embankment
- Old River and Horse Island Chute
- Row crop
- Timber and brush
- Open timber and brush
- Brush
- Pasture
- Road and embankment
- Bois Brule Levee

Table 1. Manning's n-values used in model calibration

[ft, feet]

| Land-use coverage	Lower depth		Upper depth	
	Manning's n	Depth (ft)	Manning's n	Depth (ft)
Channel and bank				
Mississippi River main channel	0.025	15	0.031	20
Spur dikes (on main channel banks)	.500	20	.050	25
Commercial and railroad embankment	.075	3	.050	8
Old River and Horse Island Chute	.040	2	.045	5
Floodplain				
Row crop	.100	3	.065	8
Timber and brush	.160	2	.220	5
Open timber and brush	.075	5	.120	10
Brush (small willows, scrub)	.170	5	.130	8
Pasture (native grass)	.055	1	.045	2
Road and embankment	.045	1	.040	2
Bois Brule Levee (crab grass, bluegrass)	.055	1	.045	3
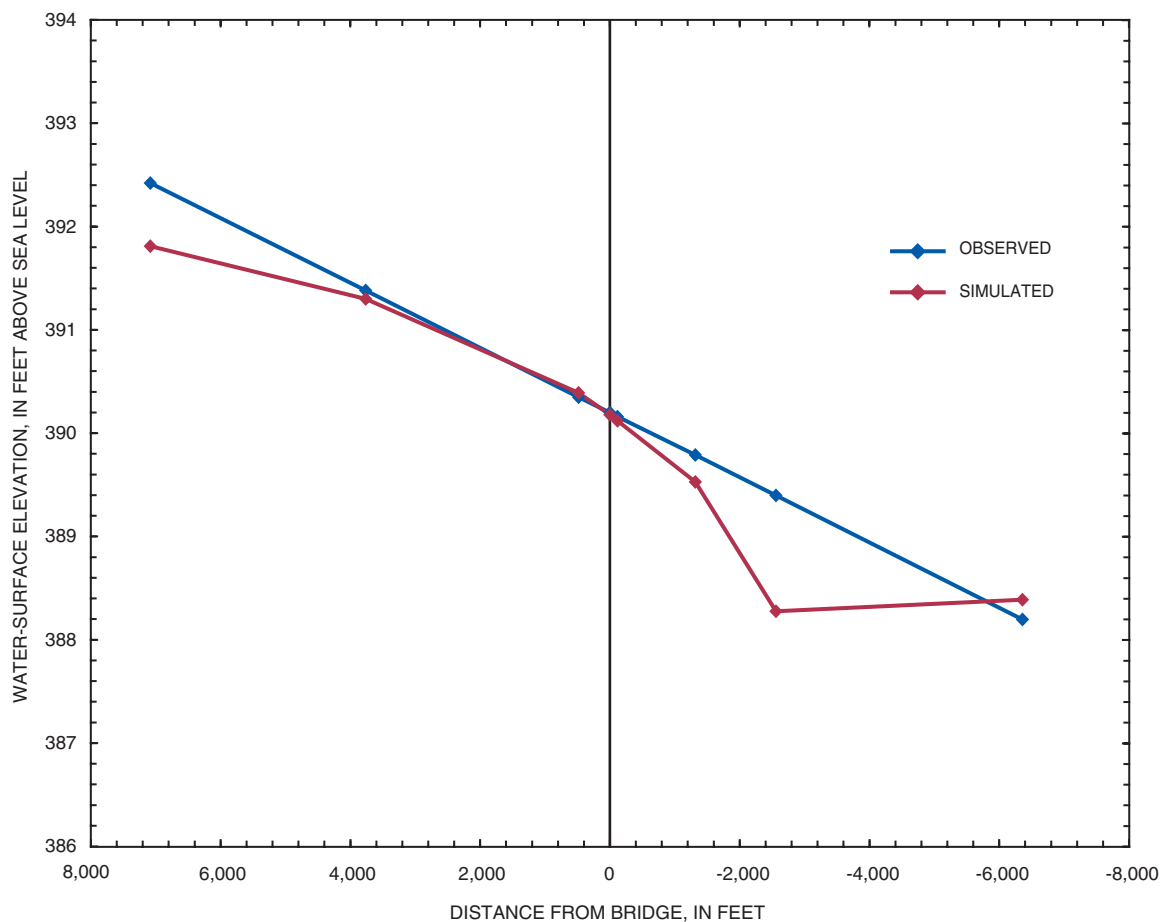
Figure 6. Observed and simulated water-surface elevation profiles for the measured discharge of 944,000 cubic feet per second along the railroad embankment on the Illinois side of the Mississippi River near Chester, Illinois.
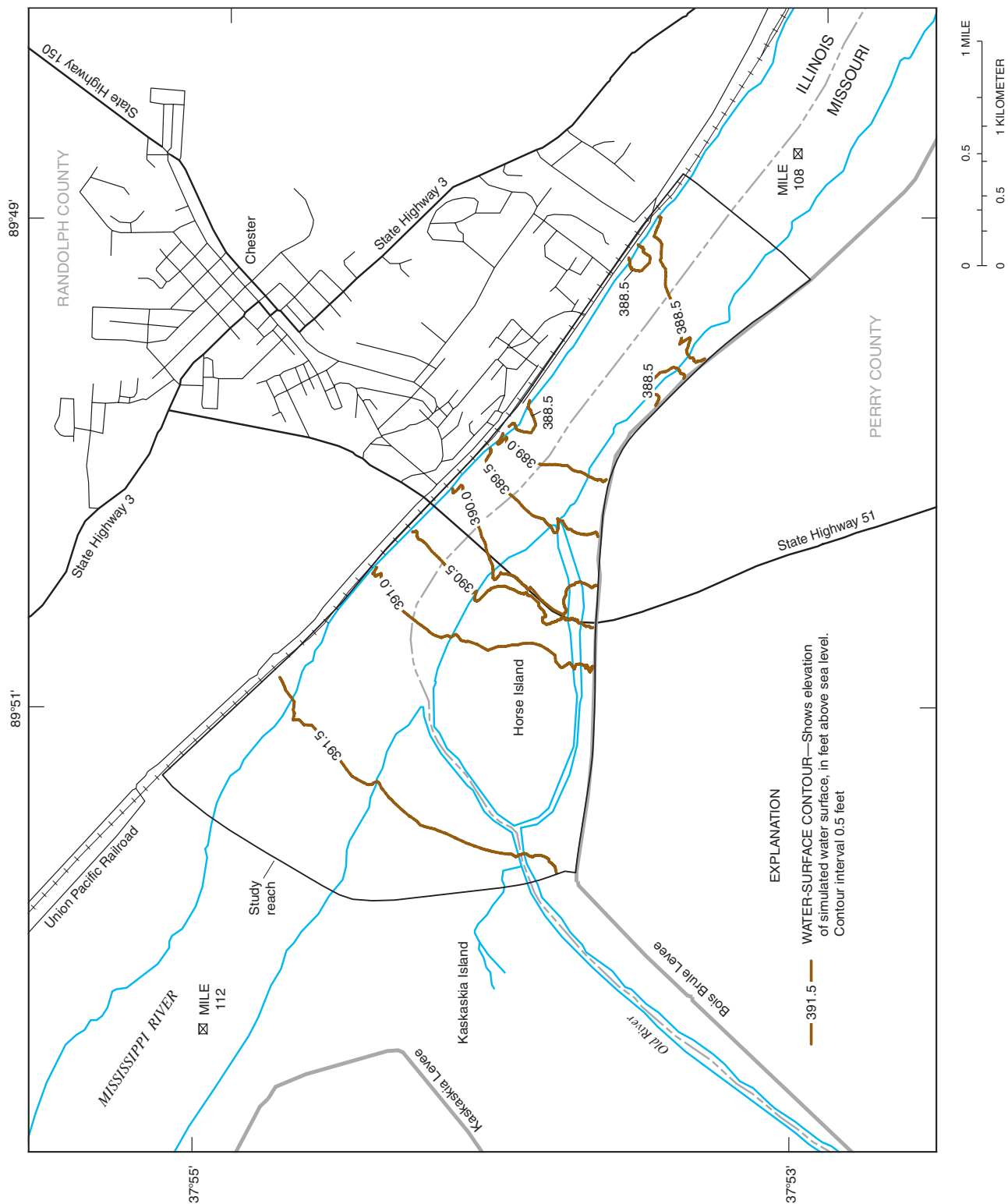
Figure 7. Simulated water-surface elevations for measured discharge of 944,000 cubic feet per second near Chester, Illinois.
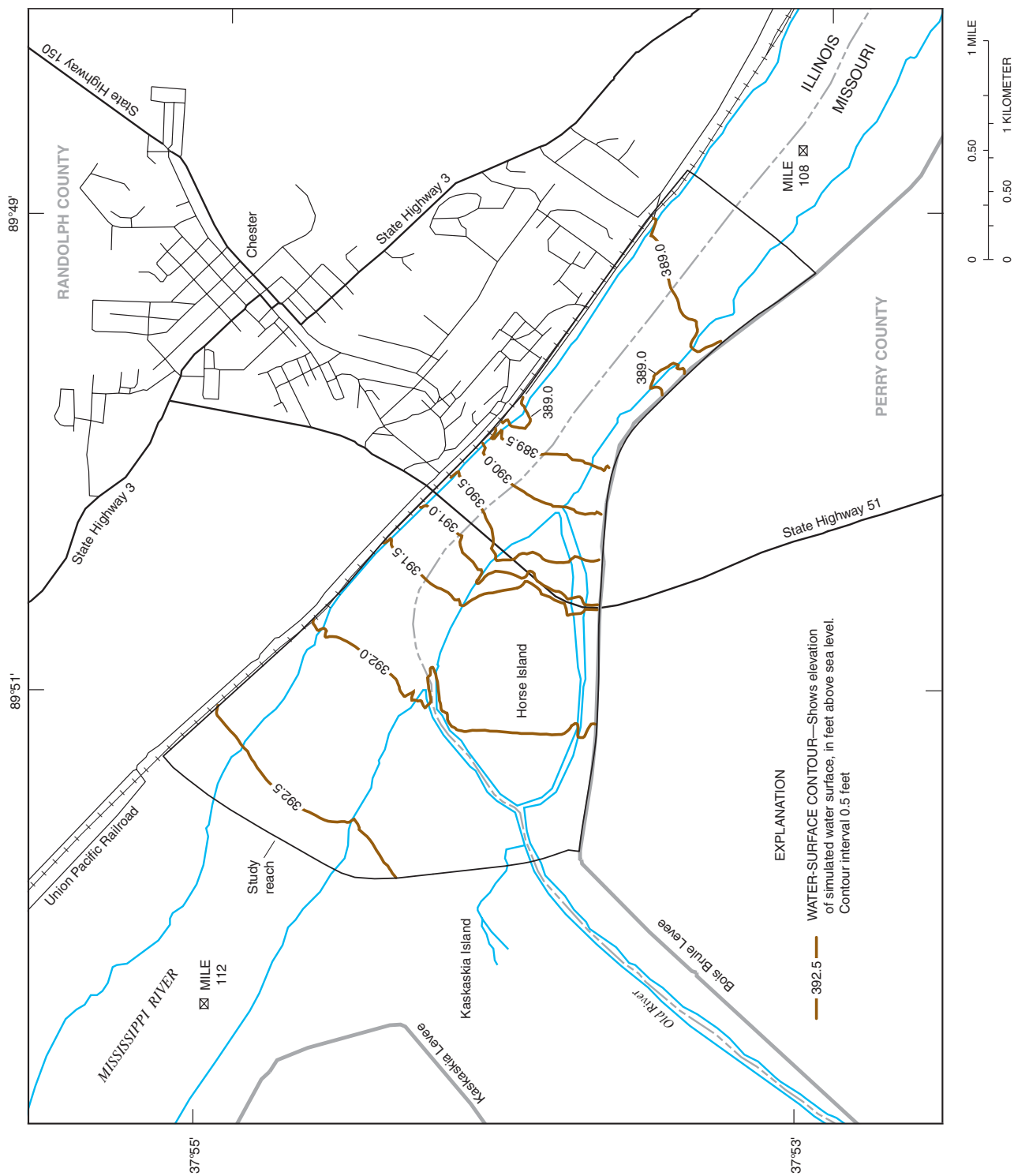
Figure 8. Simulated water-surface elevations for maximum discharge of 1,000,000 cubic feet per second near Chester, Illinois.

Horse Island Chute bridge and over the road embankment between the bridges was 63,500 ft³/s (6.4 percent of the maximum discharge).

Simulation results also indicated that the maximum velocity in the Mississippi River was 12.1 ft/s and occurred at the levee constriction; the maximum depth was 80.0 ft and occurred near the Mississippi River bridge. The maximum velocity and depth in Horse Island Chute were 4.5 ft/s and 41.0 ft, and occurred near the bridge. The maximum velocity and depth over the road embankment were 2.5 ft/s and 4.6 ft. The simulated water-surface elevation at the gage was 390.83 ft, which compares favorably with the gaging station elevation of 390.79 ft measured at the peak.

Simulation of Impendent Discharge

The impendent discharge was determined by a step process. As a starting point, the average depth of flow over the road in the calibration simulation (3.6 ft) was subtracted from the water-surface elevation at the gaging station, which resulted in an approximate elevation of 386.60 ft at the gaging station for the impendent discharge. This gage elevation, related to a discharge based on the stage-discharge relation of the gaging station, corresponded to a discharge of approximately 880,000 ft³/s for the upstream boundary condition for the first step in the impendent discharge simulation.

The final results of the measured discharge simulation were used as the hot start for the first step of the impendent discharge simulation. As the tailwater elevation (the downstream boundary condition) was incrementally lowered in the spindown process, the elements along the top of the road embankment began to go dry as more of the embankment was extended above the simulated water surface. The tailwater elevation was lowered until most of the elements at the top

of the road embankment were dry, except those immediately adjacent to the Horse Island Chute bridge, which remained wet because of the backwater caused by the contraction at the bridge.

The simulated water-surface elevation at the gaging station under the conditions of the first step (387.08 ft) was related to a new discharge of 894,000 ft³/s for the second step in the impendent discharge simulation. The final results of the first step were used as the hot start for the second step, and the tailwater elevation again was lowered until the elements immediately adjacent to the Horse Island Chute bridge went dry.

The simulated water-surface elevation at the gaging station under the conditions of the second step changed by 0.01 ft from the results of the first step, so the results of the second step were assumed to be an accurate simulation of the impendent discharge. Therefore, the impendent discharge was 894,000 ft³/s with a water-surface elevation of 385.15 ft at the downstream end of the study reach. These simulated results compare reasonably with the corresponding parameters from the measured discharge and maximum discharge simulations, as shown in table 2.

Simulated water-surface elevations for the impendent discharge simulation are shown in figure 9. The results indicate the discharge through the Mississippi River bridge was 846,000 ft³/s (94.7 percent of the impendent discharge), and discharge through the Horse Island Chute bridge was 47,800 ft³/s (5.3 percent of the impendent discharge). Simulation results also indicated that the maximum velocity in the main channel was 11.7 ft/s and occurred at the levee constriction; the maximum depth was 76.3 ft and occurred near the Mississippi River bridge. The maximum velocity and depth in Horse Island Chute were 4.2 ft/s and 37.2 ft, and occurred near the bridge.

Table 2. Final simulated results for three alternative discharges

[ft³/s, cubic feet per second; ft, feet]

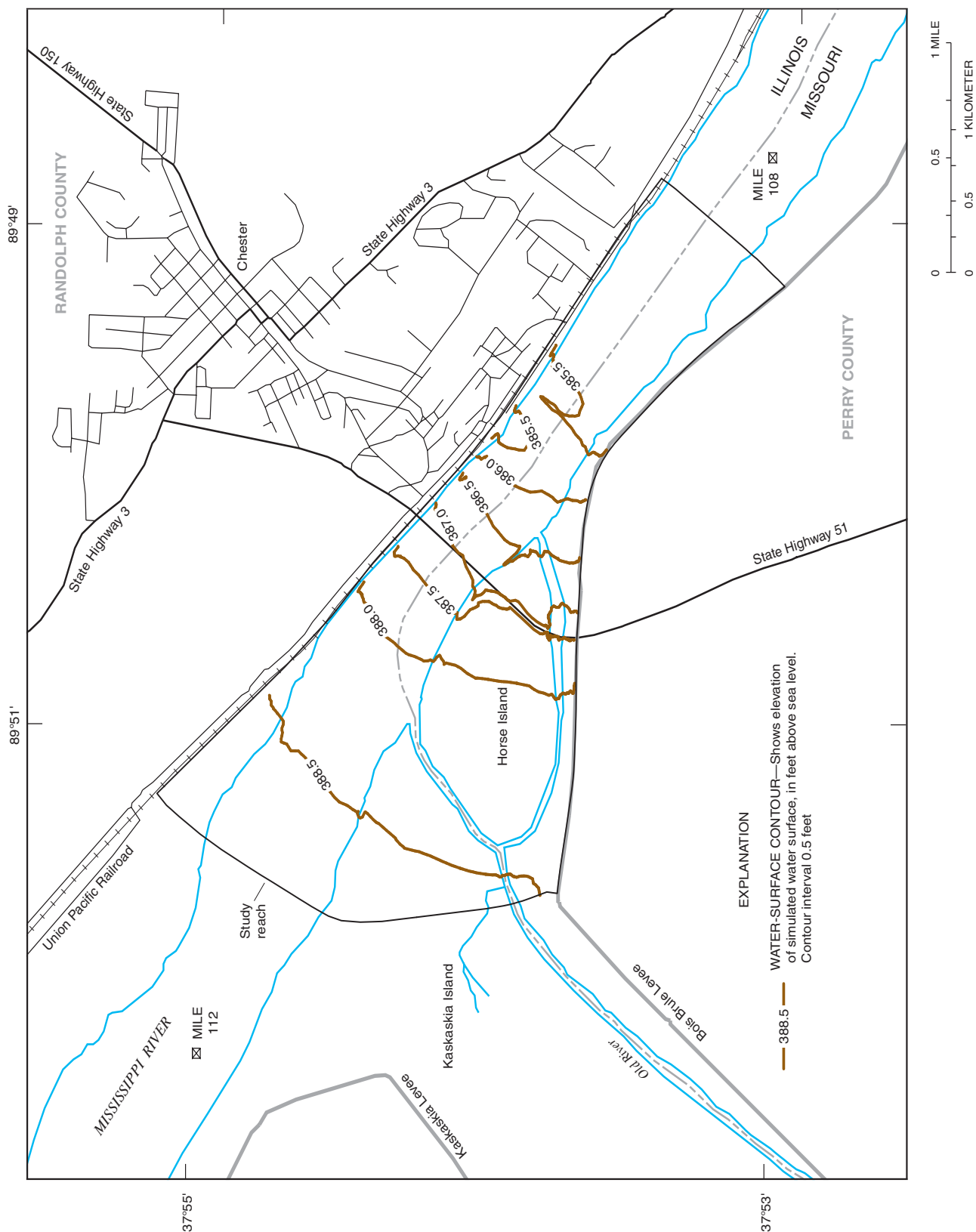| Discharge type	Discharge (ft³/s)	Water-surface elevation at gage (ft)	Water-surface elevation at downstream boundary (ft)	Difference in water-surface elevations (ft)
Measured	944,000	390.18	388.35	1.83
Maximum	1,000,000	390.83	388.85	1.98
Impendent	894,000	387.09	385.15	1.94

Figure 9. Simulated water-surface elevations for impendent discharge of 894,000 cubic feet per second near Chester, Illinois.

EXPLANATION

WATER-SURFACE CONTOUR—Shows elevation of simulated water surface, in feet above sea level. Contour interval 0.5 feet

— 388.5 —

Historical records at the Mississippi River gaging station show that the discharge measured through the Horse Island Chute bridge is related to the total discharge measured for the Mississippi River at the gaging station. A graph of discharge measurements through Horse Island Chute (fig. 10) indicates that once the total discharge measured at the the gaging station exceeds approximately 452,000 ft^3/s (with approximately a 2-year discharge recurrance interval), water in the Mississippi River main channel overtops the banks and begins to flow toward Horse Island Chute. At total discharges larger than 452,000 ft^3/s, the discharge in Horse Island Chute is approximated by the relation:

$$Q_{Chute} = 0.105 Q_{Total} - 47,500,$$ where Q_{Chute} is the discharge in the Horse Island Chute and Q_{Total} is the total discharge measured at the gaging station. As determined by the simulation, flow over the road embankment exists for total discharges greater than 894,000 ft^3/s, so the discharge in Horse Island Chute calculated using the regression equation for a total discharge above 894,000 ft^3/s will not include the quantity of discharge over the road. Using this relation, the discharge in Horse Island Chute for the three discharges was compared with the simulation results, excluding road overflow (table 3).
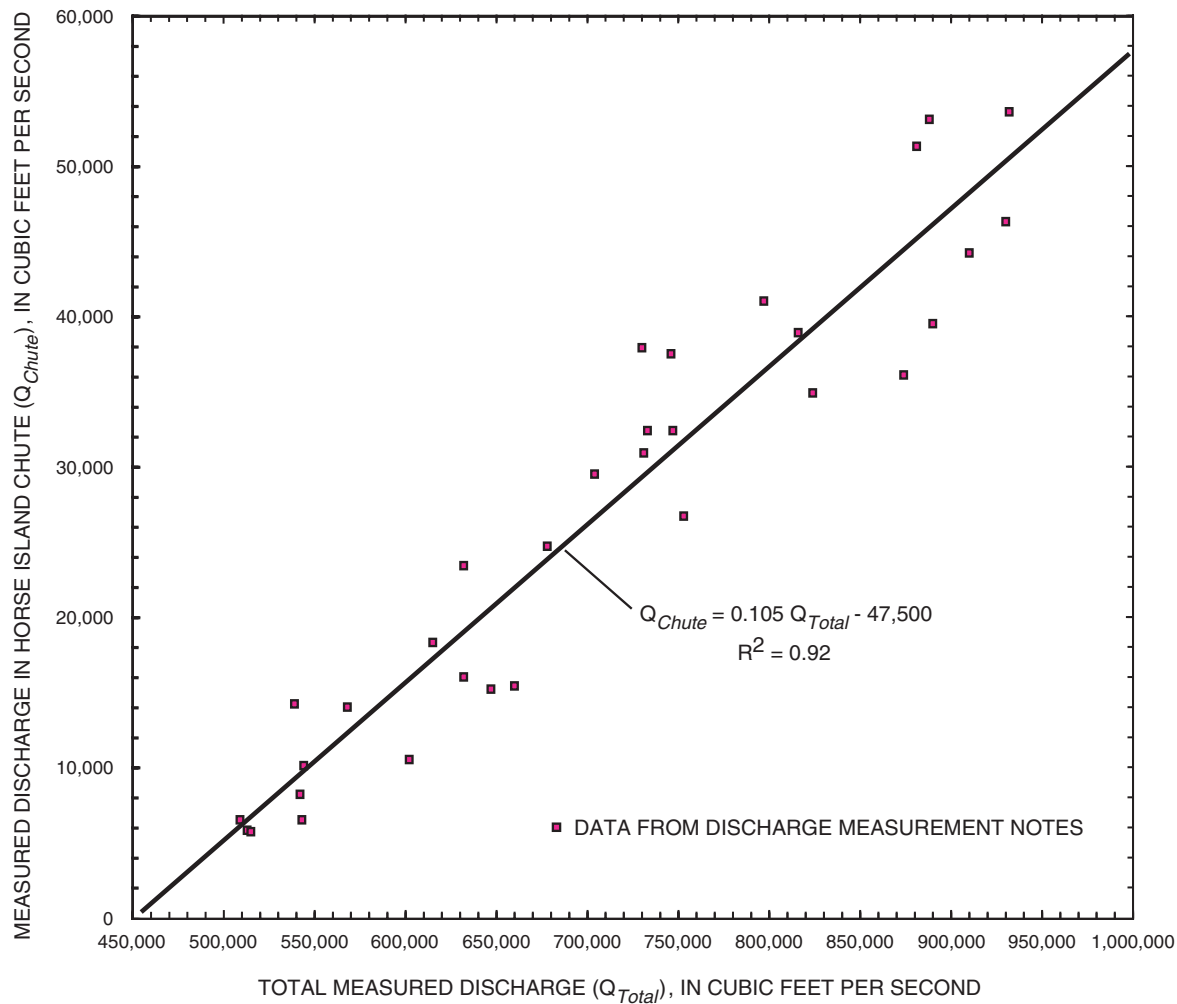

Figure 10. Relation between measured discharge in Horse Island Chute (Q_{Chute}) and total measured discharge (Q_{Total}) for Mississippi River at the U.S. Geological Survey gaging station at Chester, Illinois.

Table 3. Simulated and calculated discharges through Horse Island Chute bridge

[ft^3/s, cubic feet per second]

Discharge	Total discharge (ft^3/s)	Simulated discharge through Horse Island Chute bridge[1] (ft^3/s)	Calculated discharge through Horse Island Chute bridge based on regression (ft^3/s)	Simulated minus calculated discharge (ft^3/s)	Percent difference from calculated discharge
Measured	944,000	54,700	51,600	3,100	6.0
Maximum	1,000,000	59,200	57,500	1,700	3.0
Impendent	894,000	47,800	46,400	1,400	3.0

[1] Flow over road embankment is not included.

EVALUATION OF SCOUR

Scour is the general term used to describe the erosion of channel material and can be divided into three primary components:

• Degradation (or aggradation), which refers to the general lowering (or raising) in elevation of a channel bed with time because of sediment transport;

• General contraction scour, which describes the erosion caused by increased velocities and turbulence that occur during flooding or result from the contraction of flow by bridge approach embankments or other constrictions that encroach on a floodplain; and

• Local scour, which describes the localized removal of material by increased velocities and turbulence resulting from local disturbances to flow such as bridge piers, abutments, and spur dikes/guidebanks (Richardson and others, 1995).

Although scour processes continuously are at work, they are accelerated during high-flow conditions, thereby increasing the potential for scour-related problems at a bridge.

Using techniques outlined in Hydraulic Engineering Circular No. 18 (HEC-18), entitled "Evaluating Scour at Bridges" (Richardson and others, 1995), the various components of scour can be estimated from hydraulic parameters determined by numerical modeling and bridge parameters obtained from bridge plans and surveys taken of the site. Long-term degradation or aggradation can be determined from a comparison of the present channel cross section at the bridge with the channel cross section at the time of bridge construction. General contraction scour depths can be estimated using the ratio of the discharge in the approach channel to the discharge in the bridge opening and the ratio of the width of the approach channel to the width of the

bridge opening. Local pier scour depths can be estimated at each pier or bent using flow velocity, flow depth, pier width, pier length, and flow approach angle. Local abutment scour depths can be estimated for the approach embankment on each side of the bridge using the amount of discharge and flow area on the floodplain that is blocked by each approach embankment.

Because the Bois Brule Levee in the vicinity of Missouri State Highway 51 was not breached, the funneling effect of the levee downstream from Missouri State Highway 51 caused the velocities of water in the Mississippi River, Horse Island Chute, and in the adjoining floodplains to be greater during the 1993 flood than they would have been had the levee been breached. Therefore, the conditions during this record flood were almost ideal for determining the most extreme scour effects at the Horse Island Chute bridge. Another situation for maximum scour exists when adjacent road embankments are just about to be overtopped, as is the case in the impendent discharge scenario, because all of the flow on adjoining floodplains is forced through the bridge opening, instead of flowing directly over the roadway.

Scour Depth Computations In Finite Element Surface-Water Modeling System

The FESWMS-2DH model is able to compute general contraction scour at each node, using a critical bed shear stress (τ_0, the stress that must be overcome to move a soil particle by the flow) and various other parameters computed by the model during the simulation. Chow (1959) defines τ_0 as "permissible tractive force" or "permissible shear force" and provides several tables for estimating τ_0 based on material type, par-

ticle size, and cohesion. Furthermore, Froehlich (1989) provides guidance for estimating τ_0 based on soil cover. However, the critical bed shear stress was difficult to estimate with confidence. The critical bed shear stress for the Mississippi River was estimated as 0.09 lb/ft^2 (pounds per square foot) based on the median particle diameter (D_{50}, the particle diameter at which 50 percent of the material is coarser and 50 percent is finer) of the material found in the main channel [D_{50} is 0.76 mm (millimeter)], but specific information about the channel and floodplain conditions during the flood was not available. Therefore, the general contraction scour depths computed by the model were not used.

However, the constriction in the floodplain downstream of the Mississippi River bridge caused a substantial amount of general contraction scour because of increased velocities in this part of the study reach. A contour plot of the velocity magnitudes in the study reach for the maximum discharge simulation are shown in figure 11. The velocity magnitude reaches a maximum just upstream from the point of maximum constriction by the levee and diminishes downstream. This velocity increase and subsequent downstream expansion is likely the cause of the relatively flat water-surface profiles seen at and downstream from the constriction in figures 6 through 9, because an increase in velocity results in a decreased flow depth in a sub-critical flow reach. Furthermore, general contraction scour depths computed at nodes in this part of the study reach reached 35 ft at the point of maximum velocity in the maximum discharge simulation, indicating the shelf of sediment deposited at the mouth of Horse Island Chute may not have been present in the 1993 flood. Thus, the flood-profile marks in the vicinity of the constriction are substantially higher that the simulated water-surface elevations.

Local pier scour also can be computed by the model when the user defines the location, size, alignment, and various other parameters for each pier or bent. However, the pier scour depths computed by the model are for the existing channel configuration. Currently (2001), the scour assessments performed for the MoDOT are for most extreme conditions, which allow for a dynamic channel configuration such as a shift in the channel thalweg during a flood. Therefore, the pier scour depths computed by the model were not used.

A magnification of the finite-element mesh in the vicinity of the Horse Island Chute bridge is shown in figure 12 with results from the maximum discharge

simulation. The depth of flow is indicated with shaded areas, and the velocity magnitude computed at each corner node is indicated with a vector. The velocity vectors indicate the line of separation of flow on the embankment between the two bridges occurs approximately 115 ft north of the left (north) end of the Horse Island Chute bridge for the maximum discharge.

In the impendent discharge simulation, the line of flow separation on the upstream face of the left (north) road embankment was expected to shift more to the north, toward the Mississippi River bridge, as flow over the road diminished. However, the velocity vectors for the impendent discharge simulation indicate that the flow separation line remained at approximately the same place as for the maximum discharge simulation and also the measured discharge simulation. The effects of the road embankment on the flow distribution through the Horse Island Chute bridge are minimal; therefore, the worst-case conditions for scour are not created in the impendent discharge simulation as was expected.

Scour Depth Computations Using Finite Element Surface-Water Modeling System Results

To obtain the parameters required for scour depth computations from the simulation results, flux lines were delineated at critical locations of interest in the finite-element mesh. The various node strings used as flux lines are indicated in figure 12. Flux lines were delineated at: 1) the upstream face of the Horse Island Chute bridge (upstream bridge face); 2) the downstream face of the Horse Island Chute bridge (downstream bridge face); 3) across the upstream channel of Horse Island Chute, from the top of the left (north) bank to the top of the right (south) bank, one bridge-length upstream from the bridge (approach); 4) along the upstream toe of the road embankment from the point of flow separation to the left (north) abutment toe (left embankment toe); 5) along the upstream toe of the road embankment from the right (south) abutment toe to the levee (right embankment toe); 6) along the upstream top edge of the road embankment from the line of flow separation to the left (north) end of the bridge (left road overflow); and 7) along the upstream top edge of the road embankment from the right (south) end of the bridge to the levee (right road overflow). The velocity magnitude, depth of flow, and water-surface elevation assigned to each node in the flux lines were obtained from the simulations and used to determine
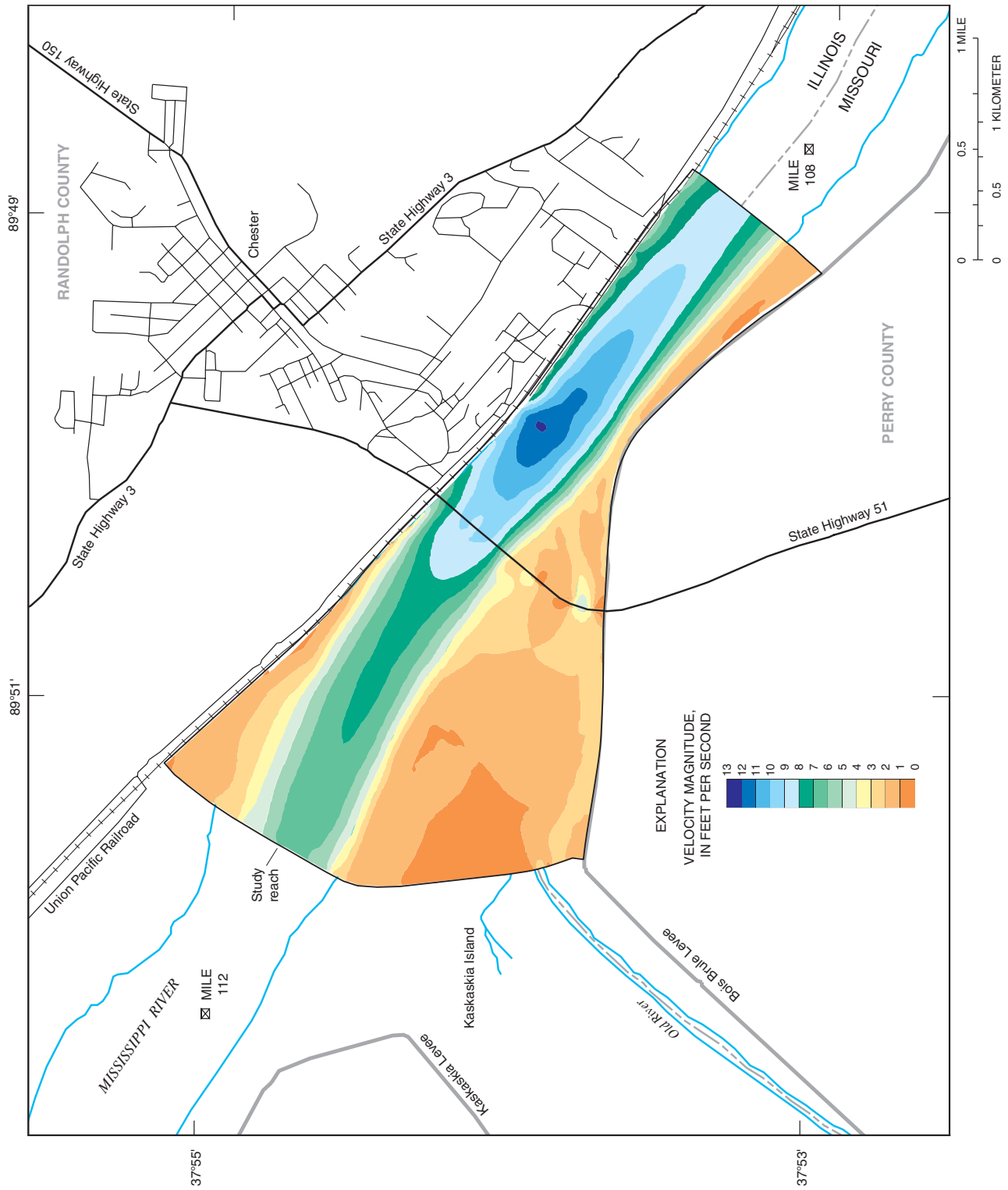
Figure 11. Simulated velocity magnitude using maximum discharge of 1,000,000 cubic feet per second near Chester, Illinois.
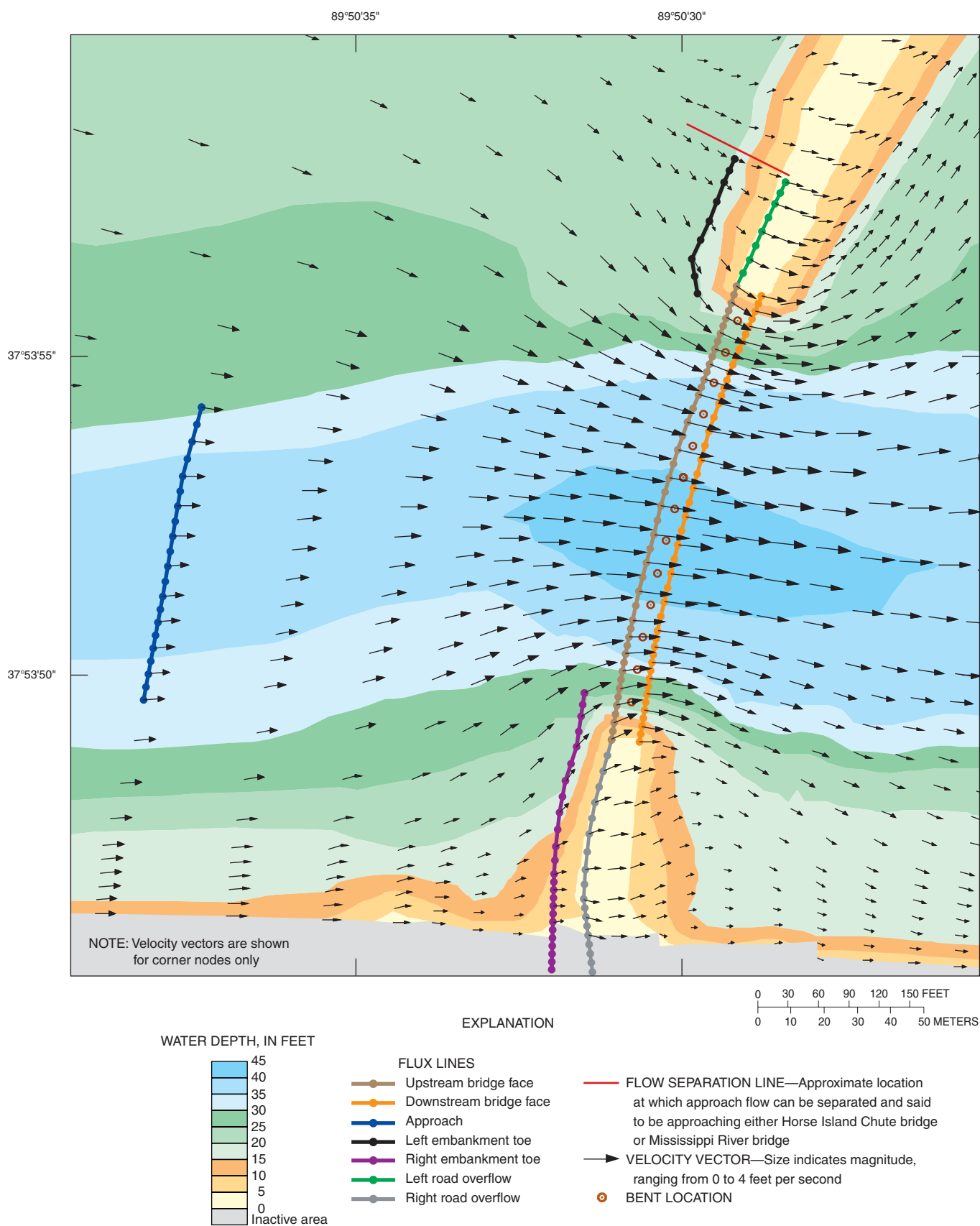
Figure 12. Simulated depths and velocity vectors for maximum discharge of 1,000,000 cubic feet per second and flux lines used for scour parameter determinations at Horse Island Chute bridge near Chester, Illinois.

the cross-sectional area of flow, average depth, and average flow velocity at each flux line. Additionally, the average elevation of the energy grade line (the sum of the land-surface elevation, the depth of flow, and the velocity head, $V^2/2g$, where V is the mean velocity of flow and g is the acceleration of gravity), was determined at the upstream flux line and the flux line at the downstream bridge face for use in the contraction scour computations.

The estimated contraction scour depths for Horse Island Chute (table 4) were calculated using Laursen's clear-water contraction scour equation (Richardson and others, 1995). Based on the velocities and depths obtained from the FESWMS-2DH analysis and the median diameter of the material in the Mississippi River (D_{50} is 0.76 mm), a critical velocity of incipient motion, V_c, for each of the three simulations was calculated using Laursen's equation (Richardson and others, 1995). Because the critical velocity was greater than the mean velocity in the approach channel for all three of the discharge simulations used in this analysis and because of the extensive vegetation found in the upstream reach, a clear-water scour scenario was assumed to exist in the Horse Island Chute channel.

The local pier scour depths (table 5) were calculated using the Colorado State University (CSU) pier scour equation (Richardson and others, 1995). Each of the 13 bents consisted of 5 octagonal concrete piles 1.334 ft in diameter. Therefore, each bent has an overall length of 6.67 ft, according to HEC-18 guidelines (Richardson and others, 1995). Each bent was analyzed using the maximum velocity magnitude and the maximum depth of flow in the channel at the upstream face of the bridge, to account for possible changes in the thalweg during a flood. The flow angle of attack as determined by FESWMS-2DH for each bent was included in the local pier scour depth computation.

The contraction and local pier scour depths computed for the maximum discharge are shown in figure 13, and the total scour (sum of contraction scour and local pier scour) for the three simulations is shown in figure 14. Scour depths are shown in relation to the land surface on January 19, 2000. The channel shows long-term aggradation, as indicated by the difference in the land surface in 1942 (from MoDOT plans) and the land surface at the time of the onsite field survey (January 19, 2000).

Table 4. Contraction scour depths and parameters used to compute scour

[ft/s, feet per second; ft³/s, cubic feet per second; ft, feet]

Parameter required to compute scour depth	Measured discharge	Maximum discharge	Impendent discharge
Critical velocity of incipient motion (ft/s)	2.69	2.70	2.65
Average velocity in approach channel (ft/s)	2.22	2.35	2.13
Type of scour	Clear water	Clear water	Clear water
Approach section (fig. 12)			
Discharge (ft³/s)	23,130	24,952	20,270
Channel width (ft)	296.6	296.6	296.6
Average depth (ft)	35.1	35.8	32.0
Energy grade line elevation (ft)	391.0	391.8	388.0
Downstream bridge face (fig. 12)			
Discharge (ft³/s)	54,830	59,297	47,797
Channel width (ft)	460.8	460.8	460.8
Average depth (ft)	33.4	34.1	30.2
Energy grade line elevation (ft)	390.5	391.2	387.4
Computed contraction scour depth (ft)	**8.1**	**10.3**	**6.7**

Table 5. Local pier scour depths and parameters used to compute scour

[MoDOT, Missouri Department of Transportation; ft, feet; ft/s, feet per second]

MoDOT bent number[1]	Stationing along centerline from left (north) end of bridge[2] (ft)	Maximum depth at upstream face[3] (ft)	Maximum velocity at upstream face[3] (ft/s)	Angle of attack computed by FESWMS-2DH (degrees)	Computed pier scour depth (ft)
		Measured discharge			
14	34.0	40.14	4.12	0.2	3.8
13	67.0	40.14	4.12	1.9	4.2
12	100.0	40.14	4.12	3.2	4.5
11	133.0	40.14	4.12	4.5	4.7
10	166.0	40.14	4.12	5.6	4.9
9	199.0	40.14	4.12	6.1	5.0
8	232.0	40.14	4.12	6.4	5.1
7	265.0	40.14	4.12	6.6	5.1
6	298.0	40.14	4.12	6.7	5.1
5	331.0	40.14	4.12	6.8	5.1
4	364.0	40.14	4.12	7.0	5.2
3	397.0	40.14	4.12	7.4	5.2
2	430.0	40.14	4.12	7.0	5.2
		Maximum discharge			
14	34.0	40.99	4.34	0.9	4.1
13	67.0	40.99	4.34	1.4	4.2
12	100.0	40.99	4.34	2.9	4.5
11	133.0	40.99	4.34	4.2	4.8
10	166.0	40.99	4.34	5.4	5.0
9	199.0	40.99	4.34	6.1	5.1
8	232.0	40.99	4.34	6.4	5.2
7	265.0	40.99	4.34	6.7	5.2
6	298.0	40.99	4.34	6.8	5.3
5	331.0	40.99	4.34	7.1	5.3
4	364.0	40.99	4.34	7.3	5.3
3	397.0	40.99	4.34	7.9	5.5
2	430.0	40.99	4.34	7.6	5.4
		Impendent discharge			
14	34.0	37.23	4.08	0.5	3.9
13	67.0	37.23	4.08	2.0	4.2
12	100.0	37.23	4.08	3.2	4.4
11	133.0	37.23	4.08	4.3	4.6
10	166.0	37.23	4.08	5.3	4.8
9	199.0	37.23	4.08	5.7	4.9
8	232.0	37.23	4.08	5.8	4.9
7	265.0	37.23	4.08	5.8	4.9
6	298.0	37.23	4.08	5.6	4.8
5	331.0	37.23	4.08	5.4	4.8
4	364.0	37.23	4.08	5.1	4.8
3	397.0	37.23	4.08	4.9	4.7
2	430.0	37.23	4.08	2.7	4.3

[1] Pier/bent number corresponds to Missouri Department of Transportation bridge plans.

[2] Stationing is the distance from the left abutment (as viewed facing downstream).

[3] Maximum depth and velocity are used for all bents to account for possible changes in the thalweg during a flood event.

Local abutment scour depths were computed for the three simulations (table 6) using the Froehlich live-bed abutment scour equation (Richardson and others, 1995). The local abutment scour depths assume no protection on the abutment faces; however, the abutment and upstream embankment faces are covered with large riprap boulders and stone revetment (fig. 15), which is expected to provide some resistance to abutment scour. Furthermore, there is no evidence of substantial scour occurring at the abutments. Therefore, no abutment scour is shown on figures 13 and 14.

Based on the computations of total scour depths (fig. 14), the scour depths indicated that the bridge bents were not substantially exposed during the maximum flood discharge on August 6, 1993. The computed contraction scour depths expose bents 5 through 10 to approximately the same depths as when the bridge was constructed in 1942. The computed scour depths for the measured discharge and the impendent discharge are less than those computed for the maximum discharge.

A channel bottom profile of Horse Island Chute at the bridge was constructed from discharge measurements obtained during the flood of 1993, using the lowest streambed elevation obtained at a given station along the bridge. This profile, shown in figures 13 and 14, indicates that the predicted scour had not occurred as of July 30, 1993, the last date a discharge measurement was obtained from the Horse Island Chute bridge before the peak on August 6, 1993.

In fact, an examination of the historical discharge measurements obtained at the Horse Island Chute bridge indicates that the channel bed was dynamic through April of 1960, but has since remained essentially stable at it's current configuration, despite several large flood events after 1960. Figure 16A shows bed profiles for small flood events, and figure 16B shows bed profiles for large flood events, the breakpoint between "small" and "large" being a flood that results in approximately 22,000 ft^3/s through Horse Island Chute bridge (approximately a 5-year recurrence interval). Before 1960, the channel bed was dynamic, with measured bed elevations as much as 13 ft lower than the as-built bed elevation during the 1943 and 1947 floods (fig. 16B). On April 11, 1960, a discharge of 24,700 ft^3/s was measured through the Horse Island Chute bridge, and the channel bed was approximately equal to the as-built bed (fig. 16B). However, on April 13, 1960, a discharge of 17,300 ft^3/s was mea-

sured through the Horse Island Chute bridge, and the channel bed had aggraded nearly 10 ft to the current bed configuration (fig. 16A).

The constriction caused by the Bois Brule Levee may have contributed to the aggradation of the channel bed at Horse Island Chute bridge. The MoDOT records for Horse Island Chute bridge indicate that the Bois Brule Levee was considerably lower when the bridge was constructed in 1942, and the levee was apparently overtopped several times in the 1940's. As a result of the large flood on the Mississippi River in 1947, the levee was raised to it's present day level during the 1940's (Francis Toohey, Attorney for Bois Brule Levee District, oral commun., 2001). The channel began to stabilize, as is shown by the bed profiles on July 10, 1951 (fig. 16A) and July 23, 1951 (fig. 16B), and reached an equilibrium after substantial sediment deposition during the flood in April 1960.

Debris was trapped on the upstream face of the bents at the time of the onsite visit (fig. 17), and measurement notes indicate that debris trapped on the upstream face of the bridge is a typical occurrence during floods. The local pier scour depths can be increased by debris lodged on the bents, but HEC-18 (Richardson and others, 1995) indicates that limited research has been done on local scour with debris and, therefore, scour equations do not include the effect of debris. The effects of debris in floods on bed profiles since 1960 apparently have been minimal (fig. 16), but additional study would be required to determine the effects of debris on the total scour that might occur at this site.

SUMMARY

A two-dimensional Finite-Element Surface-Water Modeling System (FESWMS-2DH) was used to simulate flow conditions at the Missouri State Highway 51 crossing of the Mississippi River and Horse Island Chute in Perry County, Missouri, near Chester, Illinois. The model was used to simulate flow conditions for three discharges: one simulation was for a discharge measurement made on August 5, 1993, at the time of a flood profile defined along the Illinois side of the river (measured discharge); another simulation was for the record flood peak on August 6, 1993 (maximum discharge); and the third simulation was for the discharge
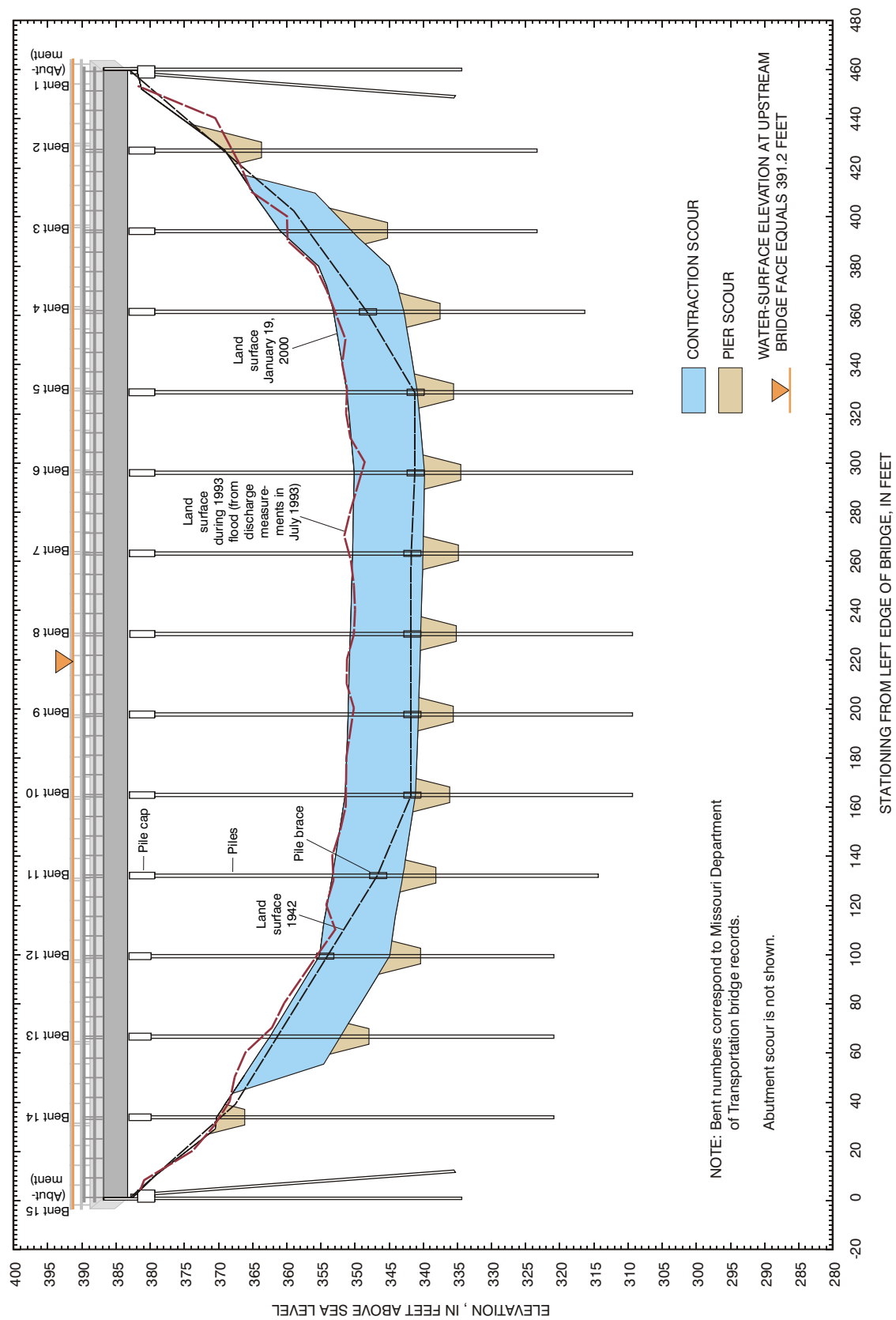
Figure 13. Contraction and local pier scour depths for simulated maximum discharge of 1,000,000 cubic feet per second through Horse Island Chute bridge near Chester, Illinois.
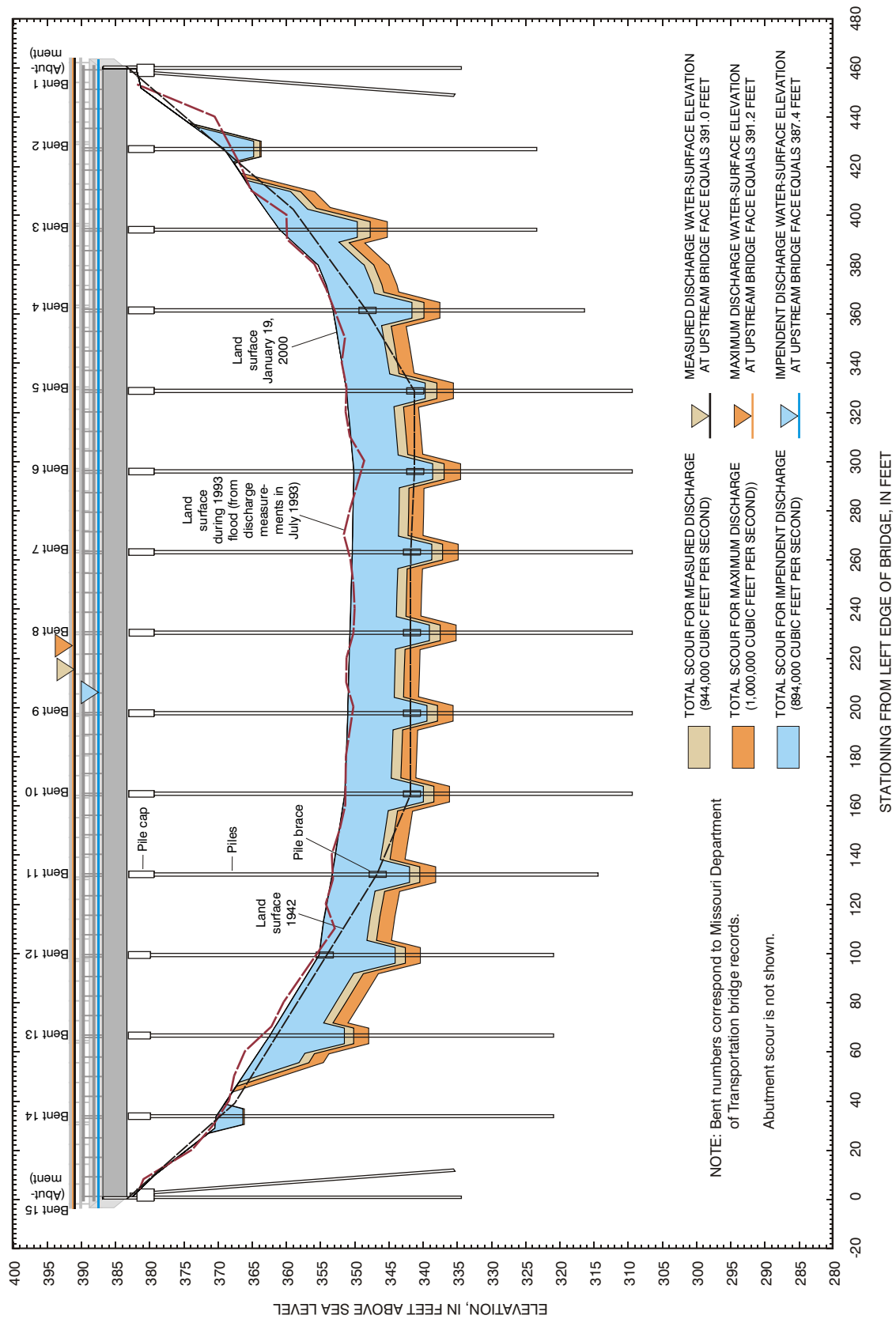
Figure 14. Total scour depths for three discharge simulations through Horse Island Chute bridge near Chester, Illinois.

Table 6. Local abutment scour depths and parameters used to compute scour

[ft³/s, cubic feet per second; ft, feet; ft², square feet; ft/s, feet per second]

Parameter required for computation of scour depth	Measured discharge	Maximum discharge	Impendent discharge
Left (north) abutment/embankment (from left abutment toe to flow separation line)			
Discharge blocked by embankment[1] (ft³/s)	1,246	1,428	715
Embankment length (ft)	143	143	143
Area of blocked flow (ft²)	2,520	2,539	2,447
Velocity at upstream toe (ft/s)	2.41	2.63	2.13
Computed abutment scour depth (ft)	**25.4**	**26.1**	**20.6**
Right (south) abutment/embankment (from right abutment toe to levee)			
Discharge blocked by embankment[1] (ft³/s)	2,928	3,337	1,160
Embankment length (ft)	278.1	278.1	278.1
Area of blocked flow (ft²)	3,140	3,178	2,998
Velocity at upstream toe (ft/s)	2.96	3.16	2.56
Computed abutment scour depth (ft)	**34.4**	**35.3**	**28.7**

[1]Discharge blocked by embankment is the difference in total approach discharge blocked and amount of flow over the road embankment.

Figure 15. Stone revetment on abutment face under left (north) end of Horse Island Chute bridge near Chester, Illinois, as viewed from downstream face of bridge.
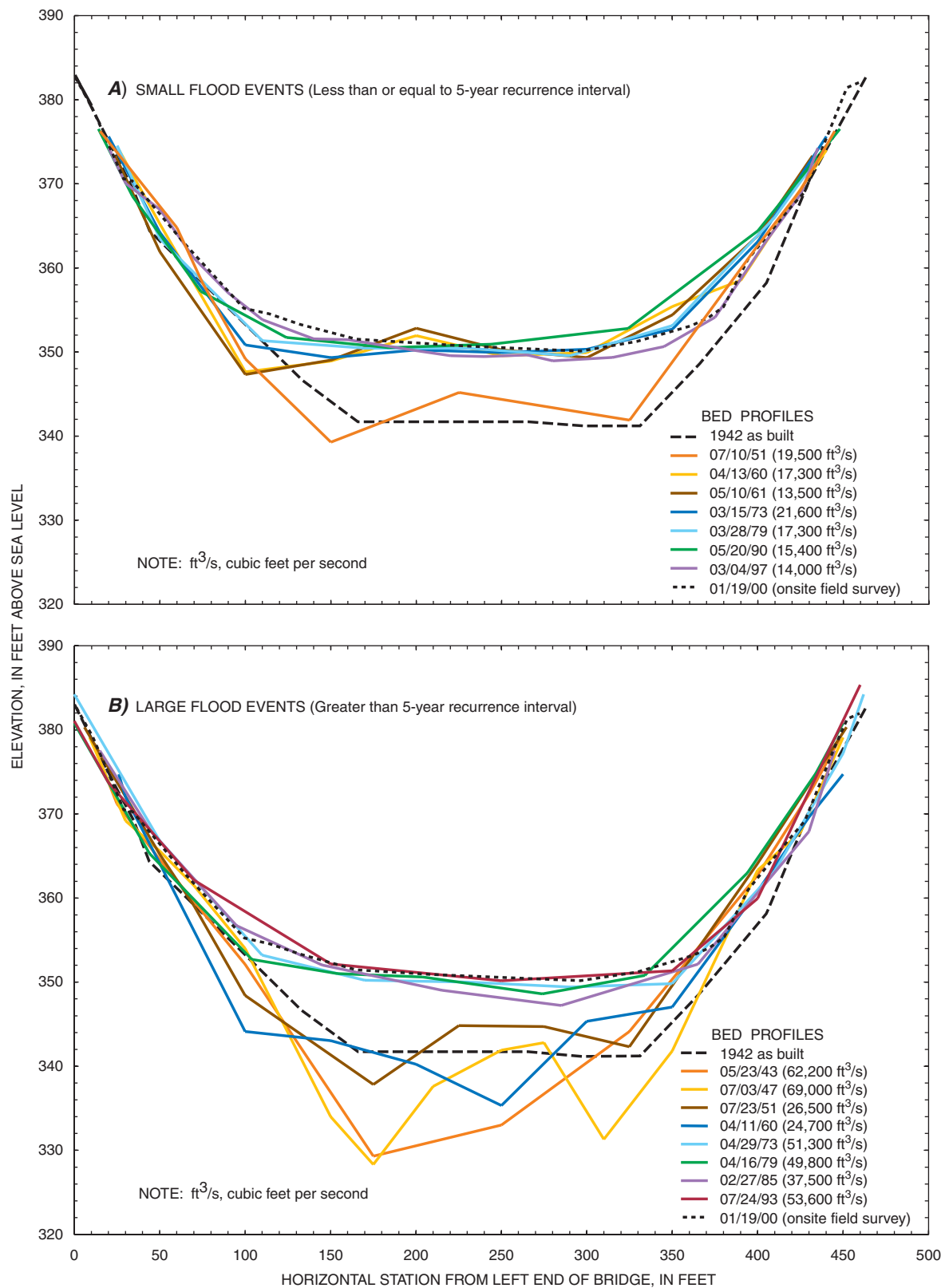
Figure 16. Channel bed profiles for; *A)* small, and *B)* large flood events at Horse Island Chute near Chester, Illinois.

Figure 17. Downstream face of Horse Island Chute bridge near Chester, Illinois, as viewed from downstream face of left (north) embankment. Note debris trapped on upstream face of bents.

that caused impending road overtopping at the road embankment in the vicinity of the Horse Island Chute bridge (impendent discharge).

Using a measured discharge of 944,000 cubic feet per second and elevation data collected on August 5, 1993, the model was calibrated to the flood profile along the railroad embankment on the Illinois side of the Mississippi River. The simulated flow conditions for the measured discharge indicated that 93.9 percent of the flow was through the Mississippi River bridge, and 6.1 percent was through the Horse Island Chute bridge and over the road embankment. The maximum velocity was 11.5 feet per second in the Mississippi River at the levee constriction, 4.3 feet per second in Horse Island Chute at the bridge, and 2.6 feet per second over the road. The simulated flow conditions for the maximum discharge of 1,000,000 cubic feet per second indicated that the percentage of flow through the Mississippi River bridge decreased slightly to 93.6 percent, with a corresponding increase to 6.4 percent through the Horse Island Chute bridge and over the road embankment. The maximum velocity increased to

12.1 feet per second in the Mississippi River at the levee constriction, and to 4.5 feet per second in Horse Island Chute at the bridge, and decreased slightly to 2.5 feet per second over the road. The impendent discharge was determined to be 894,000 cubic feet per second, with 94.7 percent of the flow going through the Mississippi River bridge, and 5.3 percent through the Horse Island Chute bridge. However, the maximum velocity decreased to 11.7 feet per second in the Mississippi River at the levee constriction and 4.2 feet per second in Horse Island Chute at the bridge.

The flow conditions determined in the simulations were used to estimate the contraction, local pier, and local abutment scour depths for the three discharges. The contraction scour depth calculated for the maximum discharge was the greatest at 10.3 feet. The contraction scour depths calculated for the measured and impendent discharges were 8.1 and 6.7 feet. The calculated local pier scour depths varied from bent to bent depending on angle of attack of flow, but the maximum pier scour depth for the maximum discharge was 5.5 feet. The maximum pier scour depths calculated for

the measured and impendent discharges were 5.2 and 4.9 feet. The maximum calculated total scour depth (contraction and local pier scour) was 13.3 feet for the measured discharge, 15.8 feet for the maximum discharge, and 11.6 feet for the impendent discharge. Abutment scour depths were calculated for the site, but are expected to be minimized by the presence of riprap boulders and stone revetment on the abutment and embankment faces.

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